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# PROCEEDINGS

## AMERICAN SOCIETY OF CIVIL ENGINEERS

NOVEMBER, 1952

# DISCUSSION OF WAVE FORCES ON BREAKWATERS

(*Published in January, 1952*)

By Kenneth Kaplan; R. G. Hennes and C. E. Leonoff; and Robert Y. Hudson

HYDRAULICS DIVISION

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## DISCUSSION

KENNETH KAPLAN.[22]—The basic homogeneous Eq. 31 for the determination of stable slopes and stone sizes of rubble-mound breakwaters, $W = \frac{K' \gamma_r \gamma_f \mu^3 H^3}{(\gamma_r - \gamma_f)^3 (\mu \cos \alpha - \sin \alpha)^3}$, may be put in the form,

$$\frac{W}{K H^3} = \frac{K'}{f(\alpha)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (33)$$

in which $K$ is a constant for any one condition of rock and fluid and $f(\alpha) = (\mu \cos \alpha - \sin \alpha)^3$. In this manner, the variation in stone weight with stable

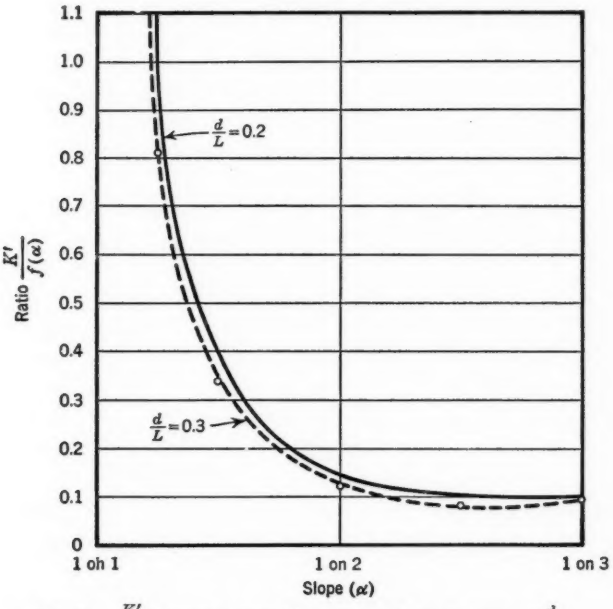


FIG. 6.—$\frac{K'}{f(\alpha)}$ VERSUS SLOPE (COT $\alpha$) FOR TWO VALUES OF $\frac{d}{L}$

breakwater slope may be found for a particular value of wave height. From the curves of Fig. 5, for any value of $\frac{d}{L}$ additional curves may be drawn showing the variation of $\frac{K'}{f(\alpha)}$ with values of $\alpha$ or slope. This has been done in Fig. 6 for $\frac{d}{L}$-values of 0.2 and 0.3.

NOTE—This paper by Robert Y. Hudson was published in January, 1952, as *Proceedings-Separate No. 113*. The numbering of footnotes in this separate is a continuation of the consecutive numbering used in the original paper.

[22] Hydraulic Engr., Beach Erosion Board, Corps of Engrs, U. S. Dept. of the Army, Washington, D. C.

The most noticeable characteristics of these curves is that $\frac{K'}{f(\alpha)}$ approaches a limiting value at an approximate slope of 1 on 2.5. This would indicate that, for any given stone weight and wave attack, there exists an optimum slope that would limit the effectiveness of a breakwater. That is, if a breakwater were built with (say) a 1 on 5 slope, it would be no more stable than one built of the same materials with a 1 on 2.5 slope.

If these conclusions are substantially correct, they could be of great value to a breakwater designer. It would be possible, for example, to determine the ultimate effectiveness of a contemplated rubble-mound breakwater in terms of the maximum stone size available at the site. Eventually it may be possible, with further study, to develop criteria for the extent of maintenance that would be required if available quarry stone is not of sufficient size to be entirely stable under expected wave attack.

Determinations such as these would be impossible if the original works of R. Iribarren and C. Nogales were to be applied, since these suggest that a breakwater may always be made stable by an increase in slope. However, before any such determinations could be made with confidence, many additional data must be gathered—specifically with view to increasing the range of $(d/L)$-values, and the range of slopes to which these conclusions may be applied.

R. G. Hennes,[23] M. ASCE, and C. E. Leonoff.[24]—A notable service to the profession is embodied in the author's critical review of classical wave theories and more recent investigations. For several years the problem of wave forces on breakwaters has been the object of experimental research under the sponsorship of the Engineering Experiment Station of the University of Washington, in Seattle. Several aspects of this study are directly related to the subject matter of the paper.

*Measurement of Wave Pressure.*—Measurements of model wave pressures on a vertical breakwater by Ben K. Rippe, J. M. ASCE, support the author's contention that the Sainflou formula is relatively reliable.[25] The tests were run in a concrete wave channel 4 ft square by 170 ft long. Waves were generated by a flexible plate so designed that its horizontal displacement at any point corresponded to the amplitude of the orbit of a water particle at that depth for a wave of specified characteristics. This wave machine,[26] produced true trochoidal waves close to the point of generation. The breakwater was a bulkhead with sides and bottom of $\frac{3}{16}$-in. sheet aluminum fastened by countersunk machine screws to $1\frac{1}{2}$-in. by $1\frac{1}{2}$-in. by $\frac{1}{8}$-in. steel angles and caulked with rubber or other type gasket material. Two separate trusses were later added to increase the rigidity. The breakwater was placed across the channel and equipped with nine pressure cells dispersed along the vertical center line, as

[23] Prof. of Civ. Eng., Univ. of Washington, Seattle, Wash.

[24] Research Fellow, Eng. Experiment Station, Univ. of Washington, Seattle, Wash.

[25] "Wave Pressures on Vertical Bulkheads," by Ben K. Rippe, thesis presented to the Univ. of Washington, at Seattle, Wash., in 1951, in partial fulfilment of the requirements for the degree of Master of Science.

[26] "Laboratory Control of Ocean Waves," by F. J. Sines, *Engineering News-Record*, Vol. 141, No. 24 December 9, 1948, pp. 96–97.

shown in Fig. 7. The cell openings are $\frac{5}{8}$ in. in diameter and well rounded on the upstream face. Rubber diaphragms about 0.03 in. thick and 2 in. square were fastened with rubber cement directly to the aluminum. Each pressure cell consisted of a $\frac{1}{2}$-in. brass disk mounted behind a rubber membrane and bolted to a thin brass beam to which were cemented two SR–4 strain gages. The cells and their electrical hookup are shown in Figs. 8 and 9. In Fig. 9, the letter A marks the position of the signal generator, B is the bridge balancing unit, C represents the pressure cell strain gages, D is a basic amplifier, E is a diode rectifier, and F is a brush magnetic recorder of two channels. The model waves had a height of 0.5 ft, a length of 7.1 ft, and a period of 1.18 sec.

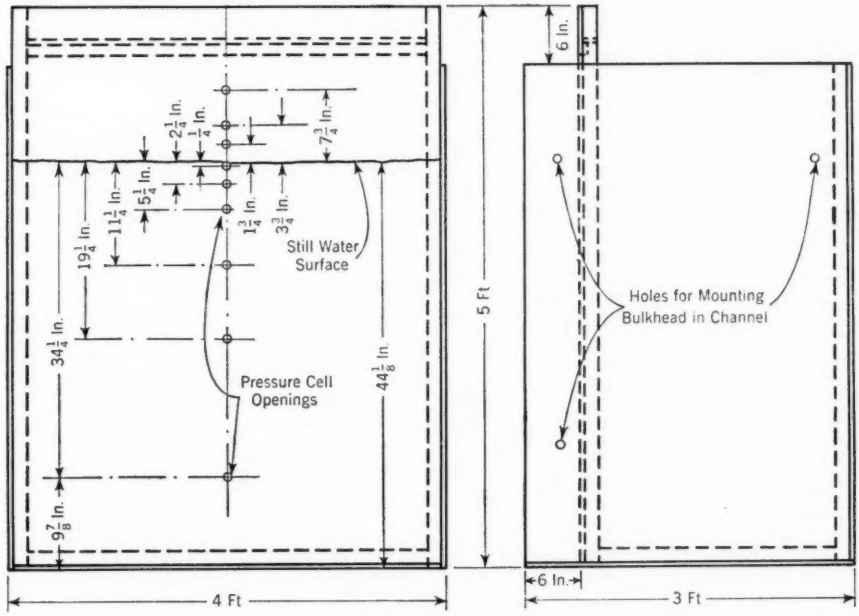


FIG. 7.—VERTICAL BULKHEAD FOR WAVE PRESSURE MEASUREMENT

The still-water depth in the channel was 3.67 ft. Thus, the $\frac{H}{L}$-ratio was 0.071 and the $\frac{d}{L}$-ratio was 0.52. The breakwater was placed at a distance of twelve wave lengths from the wave generator.

In these experiments, clapotis waves appeared at the breakwater with the arrival of the twelfth or thirteenth wave of a train. The amplitude of the ensuing waves varied over a wide range, which is contrary to the Sainflou assumption that, once formed, clapotis waves will continue to form with the same height and period, provided the generated deep-water wave does not vary.[12] Many of these waves rose considerably above the height $H + h_o$

[12] "Essai sur les diques maritimes verticales," by George Sainflou, *Annales des Ponts et Chaussées*, Paris, France, 1928. (Translated copy on file in the office of the Div. Engr., Great Lakes Div., Corps of Engrs., U. S. Army, Chicago, Ill., 1938.)

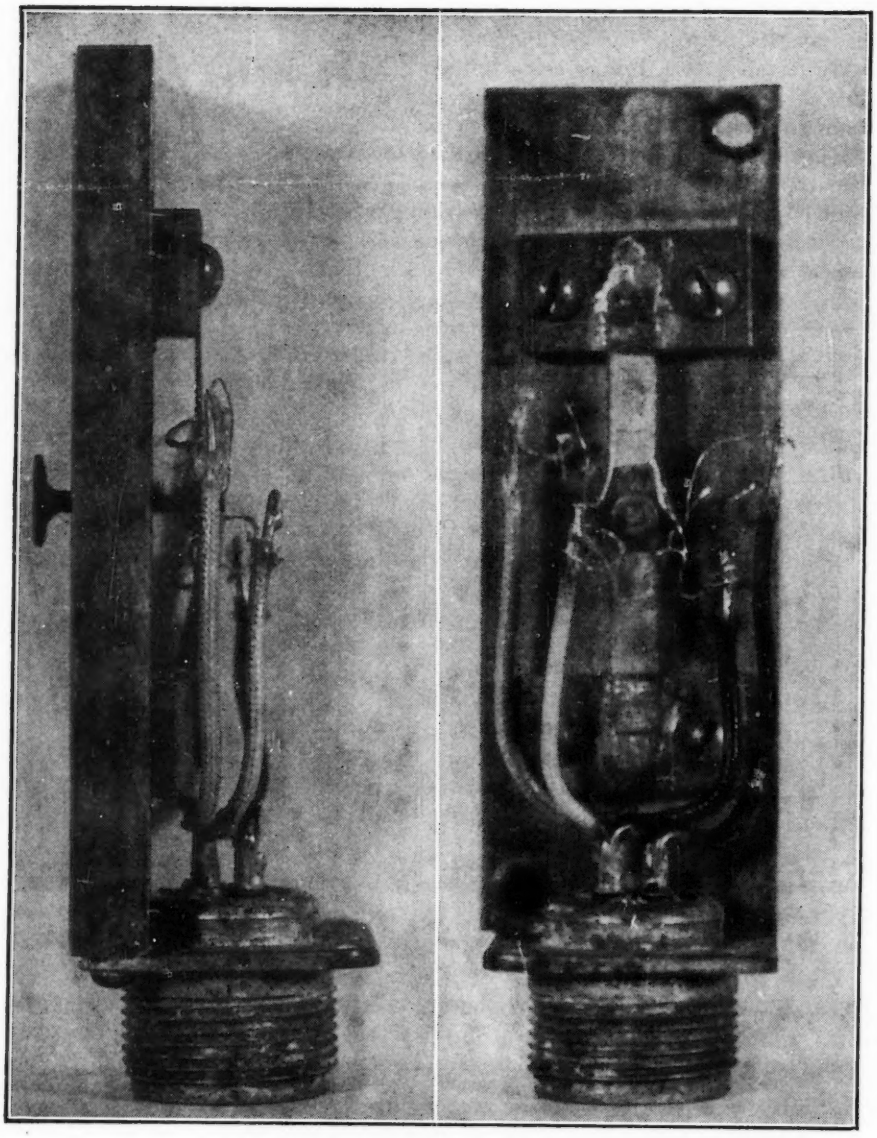

FIG. 8.—PRESSURE CELL

stipulated by Mr. Sainflou, but without any corresponding increase in maximum pressure. In fact, maximum pressure did not accompany maximum rise at the face of the breakwater. However, the evidence of these tests is that the Sainflou equation does yield the proper value for maximum pressure. This may be shown by comparing the Rippe test results with corresponding values

of unbalanced pressure (the excess of the total pressure over the static still-water pressure) obtained from the Sainflou equation (Eq. 25*a*) and the other modified wave pressure equations presented by the author (Fig. 2). Table 2 gives unbalanced pressures for a wave having the $\frac{H}{L}$ -ratios and $\frac{d}{L}$ -ratios of 0.071 and 0.52, respectively (used by Mr. Rippe), and a wave height of 12 ft, which seems to be the value used by the author in Table 1.

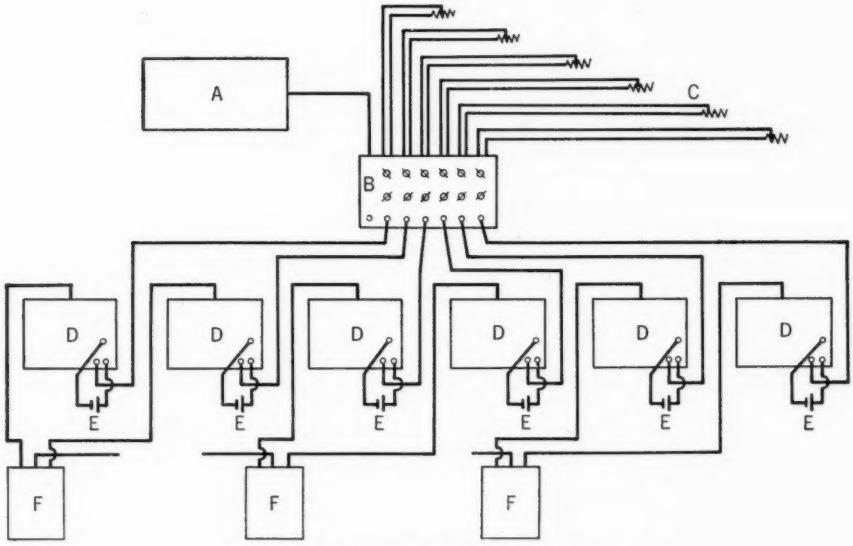


FIG. 9.—BLOCK DIAGRAM OF ELECTRICAL SYSTEM

*Experiments on Rubble-Mound Breakwaters.*—After reviewing the rather complex theories for computing wave pressures on vertical-wall breakwaters, the author presents the simpler Iribarren equation (Eq. 31) as a sufficiently accurate theoretical basis for the design of rubble-mound breakwaters. The anomaly of this situation is best exemplified by comparing the Iribarren formulas involving both vertical walls and rubble mounds. Although Eqs. 18, 19, and 20 require full knowledge of the wave characteristics, Eq. 30 involves no other property of the wave except its height, which is by no means an adequate recognition of the factors involved. The author did well to note that the Iribarren equation involving rubble mounds does not take into account the stability or instability of the attacking waves and to emphasize the need for further experimental data to evaluate the modified coefficient $K'$. (See text, under the heading, "Rubble-Mound Breakwater: Design Methods.") This is illustrated by Fig. 5, which shows that $K'$ responds so importantly to moderate variations in the $\frac{d}{L}$ -ratio. It would thus seem more appropriate to use a formula that directly recognizes wave characteristics other than $H$ alone. In an attempt to reach this objective, the writers have conducted experi-

TABLE 2.—COMPARISON OF VARIOUS METHODS OF COMPUTING UNBALANCED WAVE PRESSURES (IN POUNDS PER SQUARE FOOT)

| Depth | PRESSURES CORRESPONDING TO THE FOLLOWING VALUES OF $h_o$ (IN FEET) | | | | | | |
|---|---|---|---|---|---|---|---|
| | 0.67 | 0.67 | 0.67 | 2.7 | 0 | 1.7 | 0.80 |
| Still-water level | 585 | 789 | 688 | 794 | 660 | 1,880 | 720 |
| Mid-depth, $d/2$ | 501 | 423 | 344 | 426 | 330 | 0 | 174 |
| Authority[a] | Lira[9] | Iribarren | Benezit[11] | Saniflou[12] | Gourret[13] | Molitor[14] | Rippe[26] |

[a] References cited are those given in the text.

ments on model rubble-mound breakwaters, using the channel and wave generator previously described. Tests were run with three different generated waves as shown in Table 3. For any specified conditions, the value of $C_b$ can be read from the Hydrographic Office charts referred to by the author.[2,3] The damage to rubble-mound breakwaters is caused by breaking waves. The force of a breaking wave is proportional to the square of the particle velocity; at breaking, the orbital velocity becomes equal to the wave velocity, $C_b$, which in turn is a function of $H$, $L$, and $d$ and is a more exact criterion of the characteristics of the breaking wave than the wave height $H$ in the Iribarren formula. Hence a proper evaluation of the wave action on the rubble mounds

TABLE 3.—CHARACTERISTICS OF GENERATED TEST WAVES

| Test run | Wave height $H$, in ft | Wave length $L$, in ft | Ratio, $H/L$ | Propagation velocity $C$, in ft per sec | Wave velocity at breaking $C_b$, in ft per sec |
|---|---|---|---|---|---|
| 1 | 0.5 | 7.1 | 0.071 | 6.02 | 5.12 |
| 2 | 0.333 | 4.72 | 0.071 | 4.92 | 4.18 |
| 3 | 0.333 | 7.1 | 0.047 | 6.02 | 4.21 |

should be based on $C_b$ rather than on $H$. Only in recent years has this development been made possible through the Hydrographic Office publications. Now that maximum values of $C_b$ can be predicted nearly as well as values of $H$ for all practical purposes, it seems timely to abandon the Iribarren equation, which was handicapped by the limitations of a more primitive stage of the science, rather than to attempt to correct its defects by additional coefficients.

Beginning with a force diagram somewhat similar to that in Fig. 4, the writers developed a formula that was modified, after a study of the test data, into the equation:

$$D = \frac{0.00633\, C^2_b}{(G-1)(\tan\phi - \tan\alpha)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (34)$$

in which $D$ is the required diameter of a spherical rock, in feet; $C_b$ is the velocity of the design wave at breaking; $G$ is the specific gravity of the rock; tan $\phi$ is the

[2] "Wind, Sea, and Swell: Theory of Relations for Forecasting," by H. U. Sverdrup and W. H. Munk, Hydrographic Office, U. S. Navy, Washington, D. C., 1947.

[3] "Breakers and Surf, Principles in Forecasting," Hydrographic Office, U. S. Navy, Washington, D. C., 1944.

TABLE 4.—DIAMETER OF ROCK (IN FEET) REQUIRED FOR STABILITY UNDER WAVE ATTACK—COMPARISON OF FORMULAS

| Side slopes | Experimental | Iribarren | | de Castro y Briones | Mathews | Rodolf | Experimental | Iribarren | | de Castro y Briones | Mathews | Rodolf |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Revised | Original | | | | | Revised | Original | | | |
| | (1) | (2) | (3) | (4) | (5) | (6) | (1) | (2) | (3) | (4) | (5) | (6) |
| (a) PROTOTYPE WAVES (ASSUMPTIONS: $G = 2.70$ AND TAN $\phi = 1.09$) | | | | | | | | | | | | |
| $H_o = 50$ ft | $T = 14$ sec and $H_o/L_o = 0.0497$ | | | | | | $T = 11.7$ sec and $H_o/L_o = 0.0712$ | | | | | |
| 1 on 1.25 | 35.0 | 43.3 | 57.7 | 21.2 | 19.2 | 18.7 | 34.0 | 43.3 | 57.7 | 21.2 | 18.1 | 17.6 |
| 1 on 1.5 | 24.0 | 27.8 | 32.4 | 18.4 | 15.9 | 16.8 | 23.3 | 27.8 | 32.4 | 18.4 | 15.0 | 15.8 |
| 1 on 2 | 17.2 | 18.6 | 20.1 | 15.0 | 13.1 | 14.5 | 16.7 | 18.6 | 20.1 | 15.0 | 12.3 | 13.7 |
| 1 on 3 | 13.4 | 13.7 | 14.3 | 11.3 | 11.1 | 12.5 | 13.0 | 13.7 | 14.3 | 11.3 | 10.5 | 11.7 |
| $H_o = 35$ ft | $T = 13.8$ sec and $H_o/L_o = 0.0359$ | | | | | | $T = 9.8$ sec and $H_o/L_o = 0.0710$ | | | | | |
| 1 on 1.25 | 24.6 | 30.4 | 40.3 | 14.8 | 15.1 | 14.7 | 23.4 | 30.4 | 40.3 | 14.8 | 13.5 | 13.1 |
| 1 on 1.5 | 16.9 | 19.5 | 22.7 | 12.9 | 12.5 | 13.2 | 16.0 | 19.5 | 22.7 | 12.9 | 11.1 | 11.8 |
| 1 on 2 | 12.1 | 13.0 | 14.3 | 10.5 | 10.3 | 11.4 | 11.5 | 13.0 | 14.3 | 10.5 | 9.15 | 10.2 |
| 1 on 3 | 9.42 | 9.56 | 9.96 | 7.86 | 8.66 | 9.76 | 8.97 | 9.56 | 9.96 | 7.86 | 7.78 | 8.71 |
| $H_o = 25$ ft | $T = 11.8$ sec and $H_o/L_o = 0.0351$ | | | | | | $T = 8.0$ sec and $H_o/L_o = 0.0762$ | | | | | |
| 1 on 1.25 | 18.0 | 21.7 | 28.8 | 10.6 | 11.5 | 11.1 | 17.4 | 21.7 | 28.8 | 10.6 | 10.1 | 9.76 |
| 1 on 1.5 | 12.3 | 13.9 | 16.2 | 9.22 | 9.46 | 9.98 | 12.0 | 13.9 | 16.2 | 9.22 | 8.31 | 8.76 |
| 1 on 2 | 8.87 | 9.31 | 10.1 | 7.50 | 7.78 | 8.66 | 8.58 | 9.31 | 10.1 | 7.50 | 6.83 | 7.58 |
| 1 on 3 | 6.90 | 6.84 | 7.11 | 5.61 | 6.62 | 7.40 | 6.68 | 6.84 | 7.11 | 5.61 | 5.81 | 6.51 |
| $H_o = 15$ ft | $T = 10.8$ sec and $H_o/L_o = 0.0251$ | | | | | | $T = 6.2$ sec and $H_o/L_o = 0.0761$ | | | | | |
| 1 on 1.25 | 10.6 | 13.0 | 17.3 | 6.34 | 7.91 | 7.67 | 10.0 | 13.0 | 17.3 | 6.34 | 6.56 | 6.38 |
| 1 on 1.5 | 7.27 | 8.36 | 9.70 | 5.54 | 6.53 | 6.89 | 6.88 | 8.36 | 9.70 | 5.54 | 5.42 | 5.74 |
| 1 on 2 | 5.22 | 5.59 | 6.04 | 4.50 | 5.37 | 5.96 | 4.94 | 5.59 | 6.04 | 4.50 | 4.46 | 4.96 |
| 1 on 3 | 4.06 | 4.11 | 4.26 | 3.38 | 4.57 | 5.12 | 3.85 | 4.11 | 4.26 | 3.38 | 3.79 | 4.26 |
| $H_o = 10$ ft | $T = 9.2$ sec and $H_o/L_o = 0.0230$ | | | | | | $T = 5.1$ sec and $H_o/L_o = 0.0752$ | | | | | |
| 1 on 1.25 | 6.84 | 8.67 | 11.5 | 4.22 | 5.73 | 5.55 | 6.60 | 8.67 | 11.5 | 4.22 | 4.69 | 4.56 |
| 1 on 1.5 | 4.68 | 5.58 | 6.48 | 3.69 | 4.72 | 4.99 | 4.52 | 5.58 | 6.48 | 3.69 | 3.88 | 4.06 |
| 1 on 2 | 3.37 | 3.72 | 4.02 | 3.00 | 3.88 | 4.32 | 3.25 | 3.72 | 4.02 | 3.00 | 3.19 | 3.54 |
| 1 on 3 | 2.62 | 2.73 | 2.84 | 2.25 | 3.30 | 3.70 | 2.53 | 2.73 | 2.84 | 2.25 | 2.72 | 3.04 |
| $H_o = 5$ ft | $T = 6.5$ sec and $H_o/L_o = 0.0232$ | | | | | | $T = 3.5$ sec and $H_o/L_o = 0.0797$ | | | | | |
| 1 on 1.25 | 3.63 | 4.33 | 5.77 | 2.12 | 3.21 | 3.12 | 3.25 | 4.33 | 5.77 | 2.12 | 2.61 | 2.54 |
| 1 on 1.5 | 2.49 | 2.78 | 3.24 | 1.84 | 2.65 | 2.80 | 2.23 | 2.78 | 3.24 | 1.84 | 2.16 | 2.28 |
| 1 on 2 | 1.79 | 1.86 | 2.01 | 1.50 | 2.18 | 2.42 | 1.60 | 1.86 | 2.01 | 1.50 | 1.77 | 1.97 |
| 1 on 3 | 1.39 | 1.37 | 1.43 | 1.13 | 1.85 | 2.08 | 1.25 | 1.37 | 1.43 | 1.13 | 1.51 | 1.69 |
| $H_o = 2$ ft | $T = 4.7$ sec and $H_o/L_o = 0.0177$ | | | | | | $T = 2.3$ sec and $H_o/L_o = 0.0738$ | | | | | |
| 1 on 1.25 | 1.51 | 1.74 | 2.31 | 0.846 | 1.57 | 1.52 | 1.35 | 1.74 | 2.31 | 0.846 | 1.23 | 1.20 |
| 1 on 1.5 | 1.03 | 1.11 | 1.30 | 0.738 | 1.29 | 1.37 | 0.925 | 1.11 | 1.30 | 0.738 | 1.02 | 1.08 |
| 1 on 2 | 0.741 | 0.745 | 0.806 | 0.601 | 1.06 | 1.18 | 0.663 | 0.745 | 0.806 | 0.601 | 0.837 | 0.931 |
| 1 on 3 | 0.577 | 0.548 | 0.570 | 0.450 | 0.901 | 1.01 | 0.517 | 0.548 | 0.570 | 0.450 | 0.712 | 0.797 |

TABLE 4.—(*Continued*)

| Side slopes | Experimental | Iribarren | | de Castro y Briones | Mathews | Rodolf | Experimental | Iribarren | | de Castro y Briones | Mathews | Rodolf |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Revised | Original | | | | | Revised | Original | | | |
| | (1) | (2) | (3) | (4) | (5) | (6) | (1) | (2) | (3) | (4) | (5) | (6) |
| (*b*) Model Waves (Assumptions: $G = 2.70$ and tan $\phi = 1.171$) | | | | | | | | | | | | |
| $H_o = 0.5$ ft | $T = 1.176$ sec and $H_o/L_o = 0.0707$ | | | | | | ... | ... | ... | ... | ... | ... |
| 1 on 1.25 | 0.263 | 0.364 | 0.577 | 0.212 | 0.391 | 0.380 | ... | ... | ... | ... | ... | ... |
| 1 on 1.5 | 0.193 | 0.252 | 0.324 | 0.184 | 0.322 | 0.341 | ... | ... | ... | ... | .. | ... |
| 1 on 2 | 0.145 | 0.179 | 0.201 | 0.150 | 0.266 | 0.295 | ... | ... | ... | ... | ... | ... |
| 1 on 3 | 0.116 | 0.132 | 0.142 | 0.113 | 0.226 | 0.253 | ... | ... | ... | ... | ... | ... |
| $H_o$=0.333 ft | $T = 1.176$ sec and $H_o/L_o = 0.0471$ | | | | | | $T_o = 0.96$ sec and $H_o/L_o = 0.0706$ | | | | | |
| 1 on 1.25 | 0.178 | 0.243 | 0.384 | 0.141 | 0.298 | 0.290 | 0.176 | 0.243 | 0.384 | 0.141 | 0.278 | 0.271 |
| 1 on 1.5 | 0.130 | 0.168 | 0.217 | 0.123 | 0.246 | 0.260 | 0.129 | 0.168 | 0.217 | 0.123 | 0.230 | 0.244 |
| 1 on 2 | 0.0978 | 0.119 | 0.134 | 0.0997 | 0.202 | 0.226 | 0.0968 | 0.119 | 0.134 | 0.0997 | 0.189 | 0.210 |
| 1 on 3 | 0.0787 | 0.0884 | 0.0947 | 0.0746 | 0.172 | 0.193 | 0.0778 | 0.0884 | 0.0947 | 0.0746 | 0.161 | 0.180 |

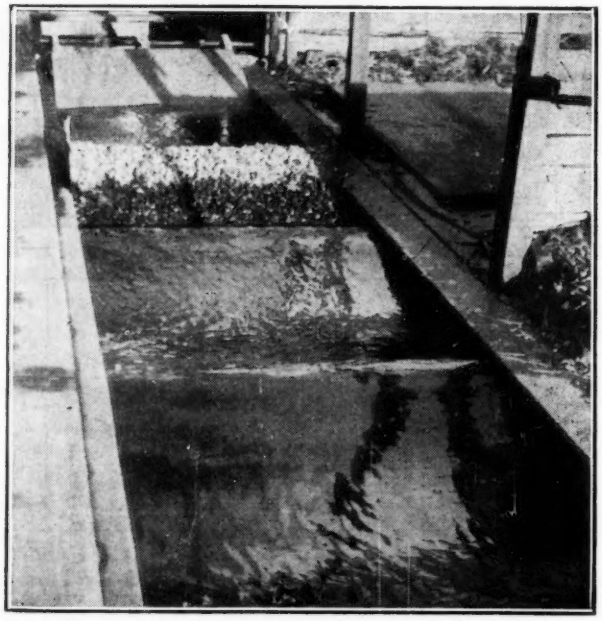

Fig. 10.—Model Wave Channel and Breakwater

coefficient of friction, rock on rock; and $\alpha$ is the angle made by the breakwater side slope with the horizontal. Table 4 presents a comparison of this formula with other published equations over a considerable range of wave heights. The revised Iribarren formula (Col. 3, Table 4) is Eq. 31. The testing program included four side slopes: 1 on 1.25, 1 on 1.5, 1 on 2, and 1 on 3. The breakwater model was constructed using crushed rock, with 71% passing the standard 1-in. sieve and retained on the $\frac{3}{4}$-in. sieve. The mean diameter of the particles was 0.925 in. This size was selected to insure turbulent seepage velocities without precluding some readjustment of the slopes. The

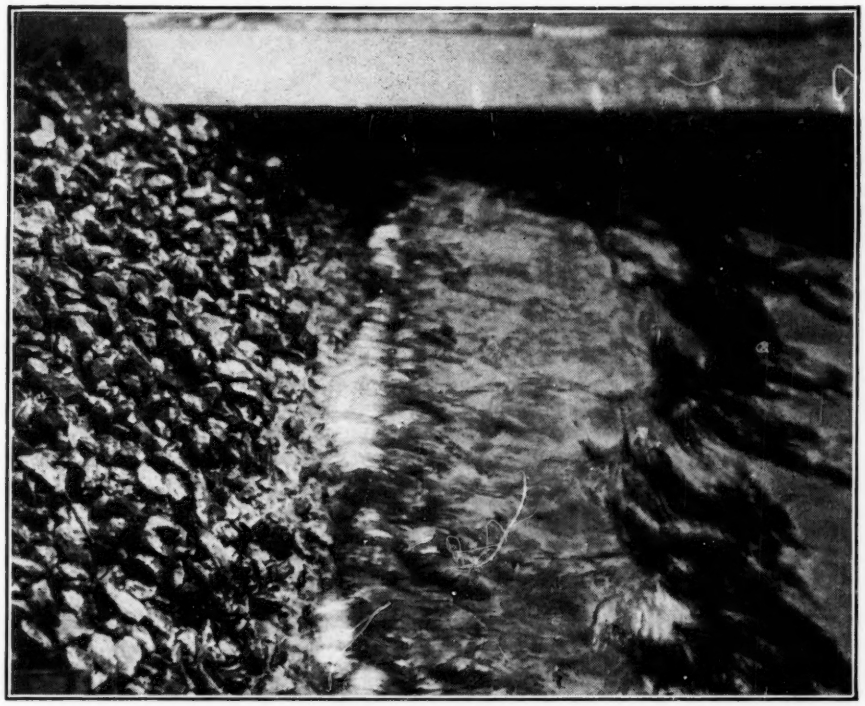

FIG. 11.—BREAKING WAVE

coefficient of friction of the crushed rock, determined by means of triaxial compression tests, was 1.171. Figs. 10, 11, and 12 provide further information on the nature of the experiment.

The author also states that tests should be performed to determine the effects of the rock shape factor, including rock shape and effective roughness. (See text, under the heading, "Rubble-Mound Breakwaters: Further Investigations.") At the University of Washington Engineering Experiment Station, a large number of direct shear tests were made on rounded river gravel and angular crushed rock, chosen to give a maximum of contrast in surface texture and angularity. The results indicate that the coefficient of friction for these two greatly diverse rock surfaces does not vary by more than about 0.035

for grain sizes between $\frac{1}{4}$ in. and $\frac{3}{4}$ in.[27,28] It would appear, therefore, that, for the type of rock normally used in rubble-mound breakwater construction, the effect of surface shape and roughness on the coefficient of friction would be insignificant in comparison to the other variables.

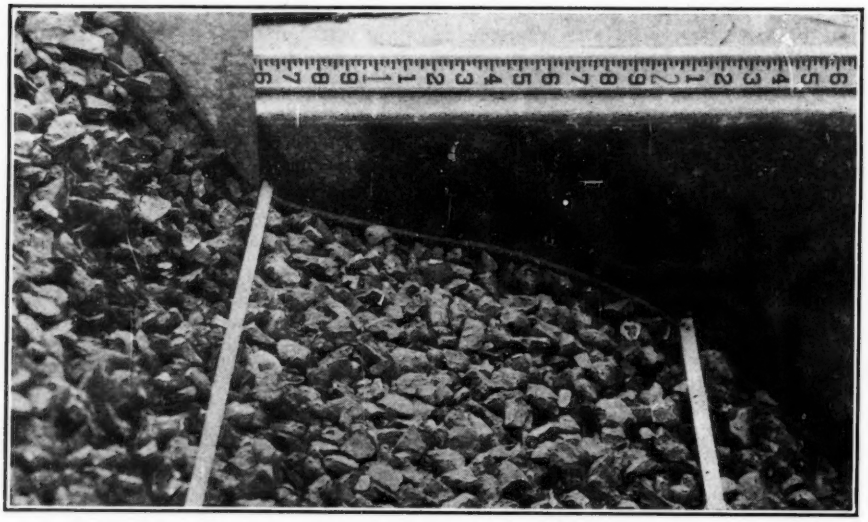

FIG. 12.—EROSION OF MODEL BREAKWATER

ROBERT Y. HUDSON,[29] M. ASCE.—The conclusions of Mr. Kaplan, concerning the significance of the $K'$-curves in Fig. 5, are correct in so far as can be determined from available test data. The curves of Fig. 5 show data from small-scale tests using one rock size, shape and specific weight, three $\frac{d}{L}$-ratios, and six side slopes. Tests completed subsequently provide data for three rock sizes, one shape factor, four specific weights, three $\frac{d}{L}$-ratios, and eight side slopes. However, the range of $\frac{d}{L}$-ratios and breakwater slopes have not been increased since 1950. It is planned to perform tests for $\frac{d}{L}$-ratios of 0.05 and 0.10 and breakwater slopes of 1 on 4 and 1 on 5.

The test data obtained since 1950 did not change the $K'$-curves appreciably from those shown in Fig. 5. Therefore, it is recommended that these curves be used for designing rubble breakwaters until more accurate and extensive information becomes available. The coefficient of friction of the rock used

[27] "Indicator Tests for Shear Resistance of Soils," by G. B. Bennett, thesis presented to the University of Washington, in Seattle, Wash., in 1949, in partial fulfilment of the requirements for the degree of Master of Science, p. 28.

[28] "The Determination of Indicative Tests for Internal Friction of Granular Soils," by H. G. Mason, thesis presented to the University of Washington, in Seattle, Wash., in 1950, in partial fulfilment of the requirements for the degree of Master of Science, p. 31.

[29] Hydr. Engr.; Chf., Wave Action Section, U. S. Waterways Experiment Station, Corps of Engrs., Dept. of the Army, Vicksburg, Miss.

in the investigation at the Waterways Experiment Station varied from about 1.01 to 1.10.

The tests performed by Messrs. Hennes and Leonoff relative to wave pressures on a vertical wall were not sufficiently extensive to determine the accuracy of the Sainflou theory. Test data for the complete ranges of the $\frac{H}{L}$-ratios and $\frac{d}{L}$-ratios possible in nature are required. Despite the statements of Messrs. Hennes and Leonoff to the contrary, the writer is still of the opinion that the generalized Iribarren formula (Eq. 31) will be sufficiently accurate for designing rubble breakwaters, when $K'$-curves which cover the complete range of variables encountered in actual practice are made available. It should be sufficient to determine $K'$-curves for $\frac{H}{L}$-ratios from 0.01 to 0.10; $\frac{d}{L}$-ratios from 0.05 to 0.50; breakwater slopes ($\alpha$) from 1 on 1 to 1 on 5; and specific gravities of cap rock, relative to pure water, from 2.0 to 3.0. The writer agrees with Messrs. Hennes and Leonoff that, for the type of rock normally used in rubble-mound breakwater construction, the effects of surface shape and roughness on the coefficient of friction are insignificant in comparison with the other variables. This applies, however, only to ordinary dumped rubble obtained from quarries. More stable breakwaters can be obtained with masonry blocks especially shaped and placed. However, this type breakwater may seldom be feasible economically.

The design formula suggested by Messrs. Hennes and Leonoff can never be made more accurate than the Iribarren formula because the principal assumption upon which it was derived is in error. Messrs. Hennes and Leonoff state: "The damage of rubble-mound breakwaters is caused by breaking waves." It was also concluded that the velocity of the wave at breaking ($C_b$) " * * * is a more exact criterion of the characteristics of the breaking wave than the wave height [$H$] of the Iribarren formula." As was explained in the paper, waves at a rubble breakwater do not necessarily break. Mr. Iribarren made an erroneous assumption similar to that of Messrs. Hennes and Leonoff when he stated that "* * * the waves break on the structure and direct jets of water downward perpendicular to the slope." This is one of the reasons $K'$-curves similar to those in Fig. 5 are necessary if the Iribarren formula is to be made sufficiently accurate for design purposes. A similar set of corrective coefficient curves would be necessary if Eq. 34 were to be made usable over the complete range of variables met in practice.

*Corrections for "Transactions."*—In lines 4 and 5 following Eq. 6, change the wording to read: "* * * the product of $y_o$ and to the specific weight of the liquid ($\gamma$) * * *"; in footnote 6, change "Blackford" to "Black" in two places; in line 6, following Eq. 28, change "0.27 $p_m$" to "0.72 $p_m$"; interchange the drawings of Figs. 3 and 4, leaving the captions as they are; in line 17, following Eq. 29, change the wording of the sentence to read: "* * * vary appreciably with the angle * * *"; in the numerator of Eq. 31 and in line 33 on page 19, change "$u$" to "$\mu$"; in the line following Eq. 32, change "a dimensionless" to "an undetermined"; and, in line 14 following Eq. 32, change "four" to "three."

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